Protocol for 100 Year Service Life of Corrugated HDPE

PART I - Evaluation and Control of Stresses in Buried Corrugated HDPE Drain Pipe

Prepared for the

Florida Department of Transportation.

By

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1 August 2003

1 August 2003

Mr. Rick Renna, Hydraulics Engineer
Florida Department of Transportation
605 Suwanee
Tallahassee, Florida 32399-0450

Project 030159 – Developing Protocol for Ensuring 100-Year Life of HDPE,
FDOT P.O. No. S5501 01783E

Dear Mr. Renna:

I am pleased to provide you with this final report on our research to establish maximum expected tension stress levels in corrugated HDPE pipe, and guidance on construction specifications to control those stresses.

I look forward to discussing our findings with you and with industry.

Sincerely yours,

Timothy J. McGrath, Principal
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ABSTRACT

The Florida Department of Transportation (FDOT) requires design of structures for a 100 year service life. This raises questions about the performance of thermoplastic pipes, which currently have 50 year properties listed in AASHTO LRFD. FDOT retained Drexel University and Simpson Gumpertz & Heger Inc. (SGH) to develop testing and analysis protocols that can be used to develop material properties and design procedures suitable for corrugated HDPE pipe for a 100 year service life. This report presents the results of the SGH work on the stress levels imposed on corrugated HDPE pipe, and construction practices to control those stresses.

A parametric analysis of buried corrugated HDPE pipe under earth loads with several compaction conditions, depths of fill, and variable support under the pipe haunches shows that long-term service tensile strain in the pipe should be less than 1.6%, corresponding to a long-term stress of approximately 320 psi. This is significantly reduced from the current AASHTO requirement of 5% long-term service tensile strain capacity. Studies on three-dimensional analysis of longitudinal strains in corrugated profiles indicate that the same minimum tensile stress capacity should also apply to longitudinal stresses. Applying a factor of safety of 1.5 to the service level stress requires that the minimum 100 year tensile strength of the pipe should be 2.5% strain, or about 500 psi.

To provide good performance and minimize the required controls on construction procedures backfill materials should be limited to well-graded, coarse-grained soils (sands and gravels) with less than 12% fines. Uniform coarse-grained soils provide good service but need to be checked for the likelihood of migration of fines into open voids. Uniform fine sands should be avoided. Coarse-grained soils with fines or fine grained soils with at least 30% coarse grained material provide good service if placed and compacted properly, but increased inspection during construction is recommended. Backfill should be compacted to at least 95% of maximum standard Proctor density for applications in roadways.

The most important aspect of construction control requires inspection of buried corrugated PE after installation. Total reduction in vertical diameter should be measured and limited to 5%. On large projects, deflections should be evaluated after a small portion of the project has been completed to determine if the construction procedures are adequate.

Minimum cover for applications subjected to live loads should be 2 ft or one-half diameter, whichever is greater.

Table of Contents

Letter of Transmittal

ABSTRACT

CONTENTS **Page**

APPENDIX

1. INTRODUCTION

As the cost of highway construction increases, transportation engineers are increasingly looking to extend the design life of highways and bridges to provide longer service without reconstruction. The AASHTO LRFD Bridge Design Specifications (AASHTO LRFD, 1998) state that the design life of bridges should be 75 years, and this code also governs the design of culverts. The Florida Department of Transportation (FDOT) requires design of structures for a 100 year service life, raising questions about the performance of thermoplastic pipes, which currently have 50 year properties listed in AASHTO LRFD. The questions pertain to the determination of long-term performance of thermoplastics, which have time dependent properties, and to the strain demand on the pipes, which is also time dependent. Of particular interest to FDOT is corrugated HDPE pipe. FDOT retained Drexel University and Simpson Gumpertz & Heger Inc. to develop testing and analysis protocols that can be used to develop material properties and design procedures that are suitable for corrugated HDPE pipe for a 100 year service life.

Issues related to time dependent material properties and material performance are addressed in reports prepared by Dr. Grace Hsuan of Drexel University. This report addresses the stress and strain demand on corrugated HDPE and on construction materials and procedures that can be used to reduce the demand.

2. ESTIMATES OF FIELD STRESS LEVELS

2.1 Considerations in Determining Stress Levels

Determining the stress level in buried pipes can be accomplished through several techniques, with varying degrees of sophistication. It has been our experience that the field control exercised during installation of buried pipes is minimal and the variability is high. Simplified design procedures are normally applied with conservative assumptions, producing designs that are adequate for typical applications. More sophisticated procedures, such as finite element analysis (FEA) are normally used only for research, large culvert sizes, or special applications where the cost of the more detailed analysis is justified through economy of fabrication or installation that can be achieved using less conservative (hence more accurate) design assumptions, and where the cost of field inspection to insure that the design assumptions are met, can be justified.

This section examines, and modifies as necessary, the simplified design procedures used by AASHTO for calculation of stress levels in buried pipe, and compares those results with predictions of FEA models. The results of these two methods are used to determine stress levels that are likely to occur in buried pipe when in service for 100 years.

This analysis is undertaken to determine the maximum tensile stress that may occur in a pipe in service for a period of 100 years. The total stress in a pipe is a combination of the bending stress that results from changes in the shape of the pipe (most commonly represented by vertical deflection), and the hoop compression stress that results from external soil loads. Total stress is most often represented as:

$$\sigma = P / A \pm M c / I \qquad \text{Eq. 2.1}$$

where:

σ = stress in pipe wall, psi

P = hoop thrust in pipe wall, lb/in.

A = cross-sectional area of pipe wall, in.^{2}/in.

M = moment in pipe wall, in.-lb/in.

c = distance from centroidal axis to extreme fiber of pipe wall, in

I = moment of inertia of pipe wall, in^4/in.

In buried pipe, the hoop thrust stress is always compression. Bending produces tension stresses on one surface and compression stresses on the other. To estimate the maximum tension stress in the pipe wall, the hoop thrust stress (P/A) is combined with the maximum tension stress produced by bending (Mc/I). It is important to recognize that:

- since tension is produced only by pipe deflection, it is important to control pipe deflection during installation,
- if the hoop thrust stress is large relative to bending, there may be no tension in the pipe, and
- the highest tension stress will occur in a shallow buried pipe (low thrust) with high deflection (high bending).

2.2 Finite Element Modeling

Finite element modeling was undertaken using the computer program CANDE. This program was developed by the US Federal Highway Administration specifically for analysis of buried pipes. The program is publicly available. The specific version of CANDE used was CANDECad. This version uses the CANDE program for calculations, but adds an Autocad based pre- and post-processor, which facilitates the modeling process.

2.2.1 FE Model

The finite element mesh used in the analysis is presented in Figure 2.1. Figures 2.2 and 2.3 show the soil zones and the construction increments used in the analysis, respectively. All analysis was completed using an embankment installation, since this generally produces more load and deflection than a trench installation.

Soil properties were those developed by Selig (1988). These properties use the hyperbolic Young's modulus developed by Duncan et al. (1980) and the hyperbolic bulk modulus developed by Selig (1988). There are three general groups of placed backfill soils in this set of properties, which are defined in AASHTO LRFD (1998) Table 12.12.2.4-2. These groups are coarse-grained soils with little or no fines (Sn), coarse-grained soils with fines or sandy or gravelly fine-grained soils (Si) and fine-grained soils (Cl). General assumptions for soils used in the analyses were:

- native soil under the pipe was considered to be a firm fine-grained material,
- a small area (Zone 5, called the "void") which is difficult to compact in the field was always considered to be filled with a very soft material (silty material at 50% of standard Proctor density, called Si50),

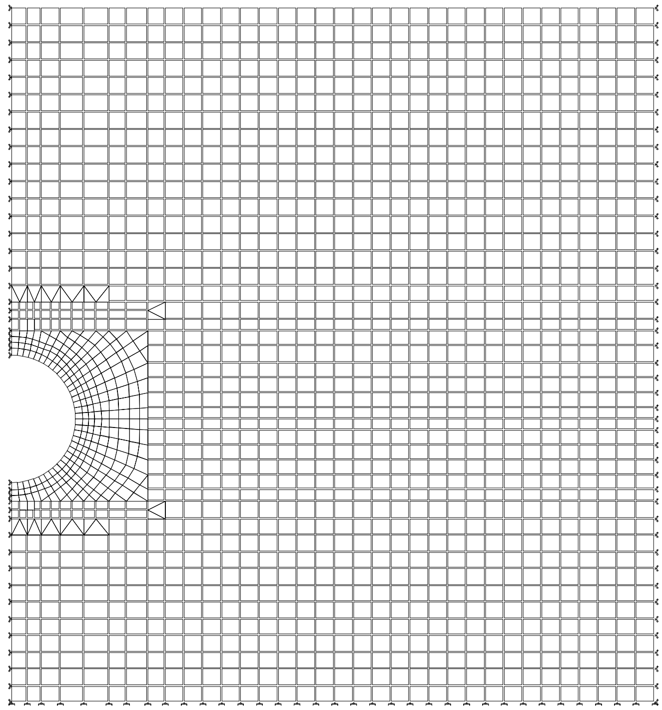

Figure 2.1 – Finite Element Mesh

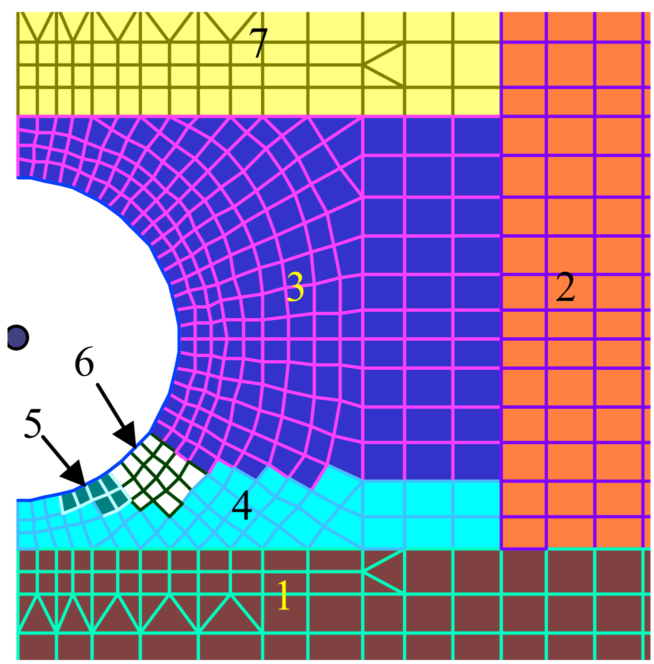

Figure 2.2 – Soil Zones

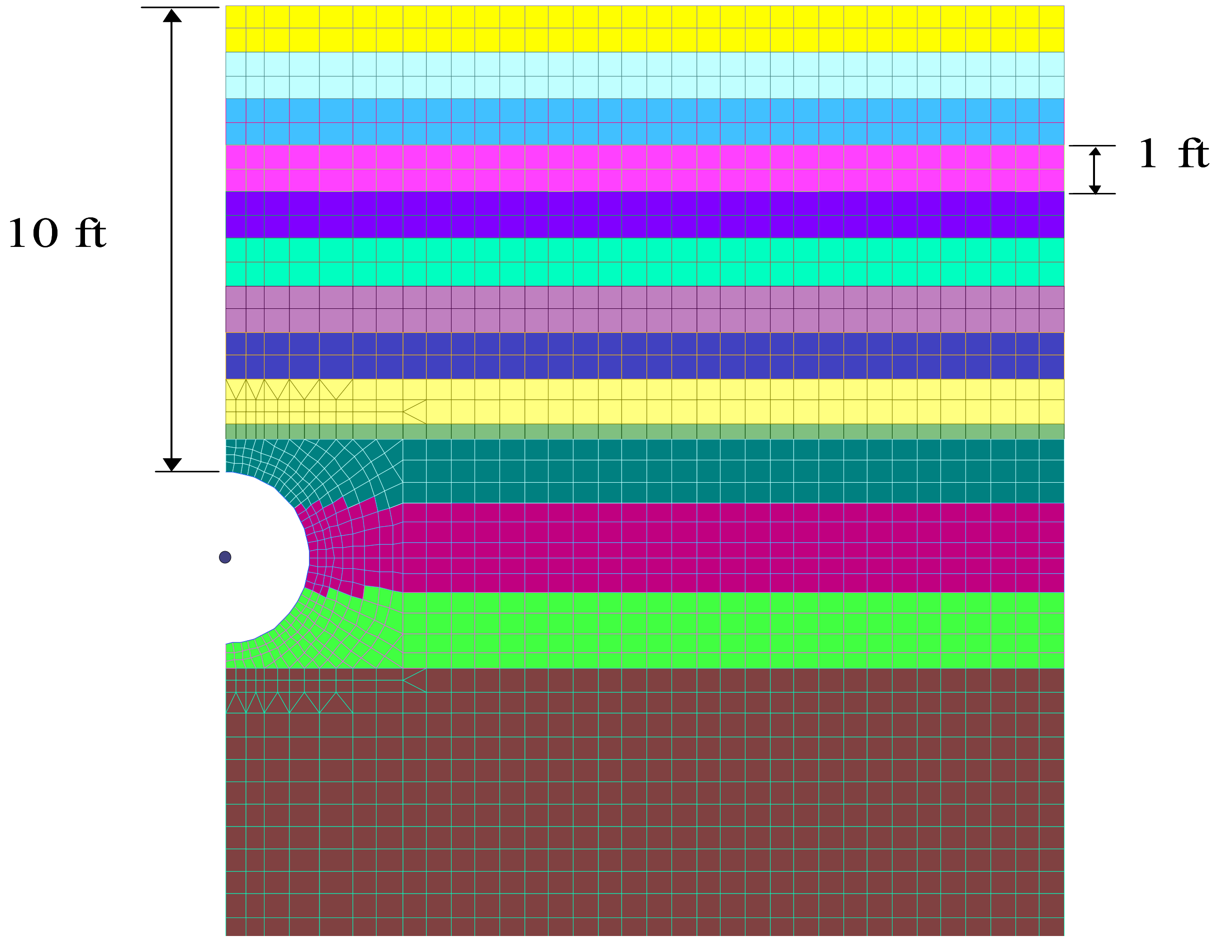

Figure 2.3 – Construction Increments

- a larger area under the pipe (Zone 6, called the "haunch zone") is considered filled with backfill soil if the pipe model is assumed to be haunched, and is filled with Si50 material if the model is assumed to have poor haunching,

- four conditions of structural backfill (Zone 3) were used; most analyses were completed with a coarse-grained material compacted to 90% of standard Proctor density (Sn90), a silty material compacted to 90% of standard Proctor density (Si90), and a silty material compacted to 85% of standard Proctor density (Si85); some analyses were also conducted with Si80 backfill; Zones 2 and 7 were modeled as structural backfill since the installations were modeled as embankment conditions, and

- the pipe bedding (Zone 4) was modeled as Si90 for all installations.

The model assumes that the pipe-soil interface was a no-slip (bonded) condition. This results in higher estimates of thrust at the springline and lower estimates of thrust at the crown than the full-slip (frictionless) condition. The AASHTO procedures for thrust design (AASHTO LRFD) are based on a mix of the no-slip and full-slip conditions (McGrath, 1999). For this study, the focus is on an estimate of the minimum hoop compression stress, thus, as noted in Section 2.3, the simplified thrust computed by AASHTO procedures should be reduced.

The coarse-grained material represents a high quality backfill with a relatively high soil modulus that can be achieved with little compactive effort. Densities of coarse-grained materials are often as high as 85% of maximum standard Proctor when dumped without compaction. The silty materials represent soils that can achieve good stiffness if compacted, but require more site control of moisture content and compactive effort. Silty soils have low stiffness if left in a dumped condition. No clay soils were considered, since these materials have marginal stiffness when compacted and require considerable controls during installation.

Analyses were completed for depths of fill from 2 ft to 21 ft.

2.2.2 Pipe Model

Most FE modeling was completed assuming a 42 in. diameter pipe. The section properties used in the analysis are presented in Table 2.1.

Table 2.1 – 42-in. Diameter PE Pipe Properties

Property	Value
Inside Diameter	42 in.
Profile Height	2.93 in.
Depth from outside surface to centroid	1.91 in.
Area	0.41 $in.^2/in.$
Moment of Inertia	0.45 $in.^2/ft$

The profile considered has the centroid eccentric from the mid-height of the profile. This produces relatively high bending strains when the pipe deflects. This condition is typical of many corrugated PE profiles available today; however, some profiles are now available with the centroid located near the mid-height of the profile and producing lower bending strains for the same deflection. The analysis results are generally applicable to all corrugated HDPE pipes.

All analyses were completed using an estimated long-term modulus of 20,000 psi, which results in good predictions of long-term thrust forces. This approach results in lower pipe stiffness during placement of backfill, but previous research, and elastic theory have demonstrated that the affect on deflection and bending is not significant.

2.3 Simplified Design Procedures

2.3.1 AASHTO Design Procedures (AASHTO LRFD Section 12.12.3.4)

Simplified analysis procedures presented here are based on the AASHTO design method for thermoplastic pipe (AASHTO, 1998) with some modifications. The AASHTO design procedure was developed to predict the maximum hoop compression in the pipe wall for the purpose of obtaining a conservative design for general and local buckling. Modifications are required to the thrust effects to predict maximum likely tension stress.

2.3.1.1 Hoop Thrust Compression Strain

Hoop thrust compression strain is computed as:

$$\varepsilon_T = 0.5\, W_{sp}\, VAF/EA \qquad \text{Eq. 2.2}$$

where:

ε_T = hoop compression strain

W_{sp} = soil prism load, lb/in

= $\gamma_s D_o (H + 0.11 D_o)$

VAF = vertical arching factor to account for pipe-soil interaction

= $0.76 – 0.71 (S_H -1.17)/ (S_H + 2.92)$

S_H = hoop stiffness factor

= $\phi_s M_s R / E A$

D_o = pipe outside diameter, in.

H = depth of fill over pipe, in.

γ_s = soil unit weight, lb/in.^{3}

ϕ_s = resistance factor to account for reduced soil stiffness, taken as 0.9

M_s = constrained soil modulus, psi, (See AASHTO LRFD, Table 12.12.2.4-1)

R = radius to centroid of pipe wall, in.

E = modulus of elasticity of pipe material, psi, taken as 20,000 psi for long-term

A = area of pipe wall, in.^{2}/in.

Two modifications were used to estimate the minimum compression stress around the pipe wall:

- to account for variation around the circumference, all thrusts were multiplied by a factor of 0.4 (see Section 2.4 Results for the basis of this)
- to account for local buckling, reduction of wall area was calculated once and not updated. The effective area varied from 0.95% of the total area at a depth of 2 ft, to 80% of the total area at a depth of 20 ft (AASHTO LRFD 12.12.3.5.3).

For shallow installations, there is some debate whether soil arching, as predicted for deep installations, will occur for shallow installations. Since the need is to predict the minimal possible thrust, arching is considered as it reduces the load on the pipe.

2.3.1.2 Bending Strain (AASHTO LRFD 12.12.3.5.4b)

Bending strain was approximated using the AASHTO equation:

$$\varepsilon_B = D_f (\Delta_b/D) (c/R) \qquad \text{Eq. 2.3}$$

where:

ε_B = bending strain in pipe wall

D_f = shape factor to account for distortion during installation, taken as 4.0

Δ_b/D = pipe deflection due to bending, expressed as a ratio to the pipe diameter to the centroid of the pipe wall

c	=	distance from centroid of pipe wall to extreme fiber of pipe wall, in., (use c_{in} or c_{out} as appropriate to calculate tension stress)
R	=	radius to centroid of pipe wall, in.

Total deflection, expressed as a ratio of the change in vertical diameter to the inside diameter, is the sum of the hoop compression strain and the vertical bending deflection:

$$\Delta_T/D = \Delta_b/D + \varepsilon_T \qquad \text{Eq. 2.4}$$

Since field control is based on total deflection, the approach taken in computing bending strain was to:

1. select a target deflection at a given depth of fill,
2. compute expected hoop thrust strain at that depth,
3. compute bending deflection by subtracting the hoop thrust strain from the target deflection, and
4. compute the bending strain based on the bending deflection.

The calculations presented in the subsequent sections are all based on total deflection, which is the sum of the hoop thrust strain and the bending strain due to deflection.

A sample calculation is presented in Attachment A.

2.4 Results

2.4.1 Effect of Backfill Material and Haunch Support

For the three backfill materials, Figs. 2.4 and 2.5 show the finite element predictions for deflection and hoop thrust versus depth of fill. This demonstrates the significant loss of stiffness as the backfill has more fines (i.e. silty, Si, versus coarse-grained soil, Sn), and/or less compaction. The thrust in the Si85 soil is almost twice that of the pipe in Sn90 soil and the deflection in the pipe in Si85 soil is more than twice that of the pipe in Sn90 soil. The pipe in Si90 soil shows intermediate results.

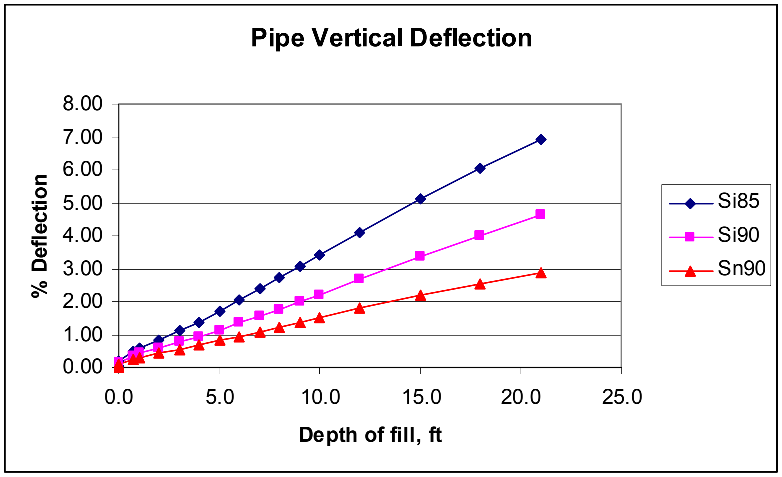

Figure 2.4 – Deflection versus Depth of Fill, Soil Type and Compaction

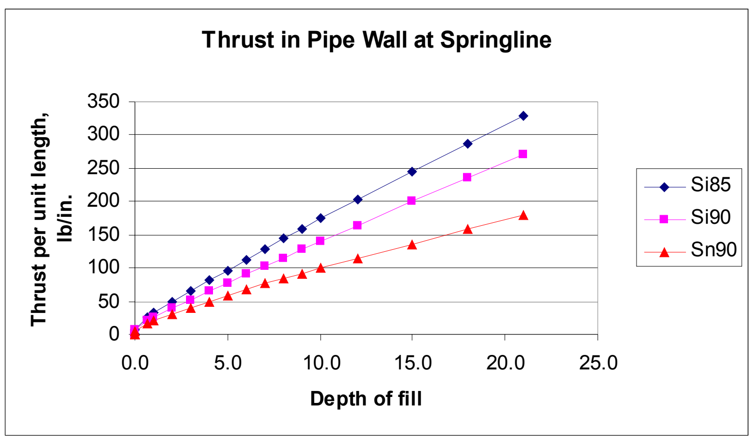

Figure 2.5 – Thrust in Pipe Wall versus Depth of Fill, Soil Type and Compaction

Fig. 2.6 shows the maximum tension bending strains at 3% deflection for pipe in different backfills and with and without haunching support. The figure demonstrates:

- at a given deflection the bending strains are quite similar regardless of the type of backfill if the support conditions are the same.
- haunch support substantially reduces the peak bending strains,
- haunch support only affects bending strain in the invert region, and

- the depths at which 3% deflection occurs varies widely, from 6 ft for a Si80- backfill to 21 ft for a Sn90 backfill.

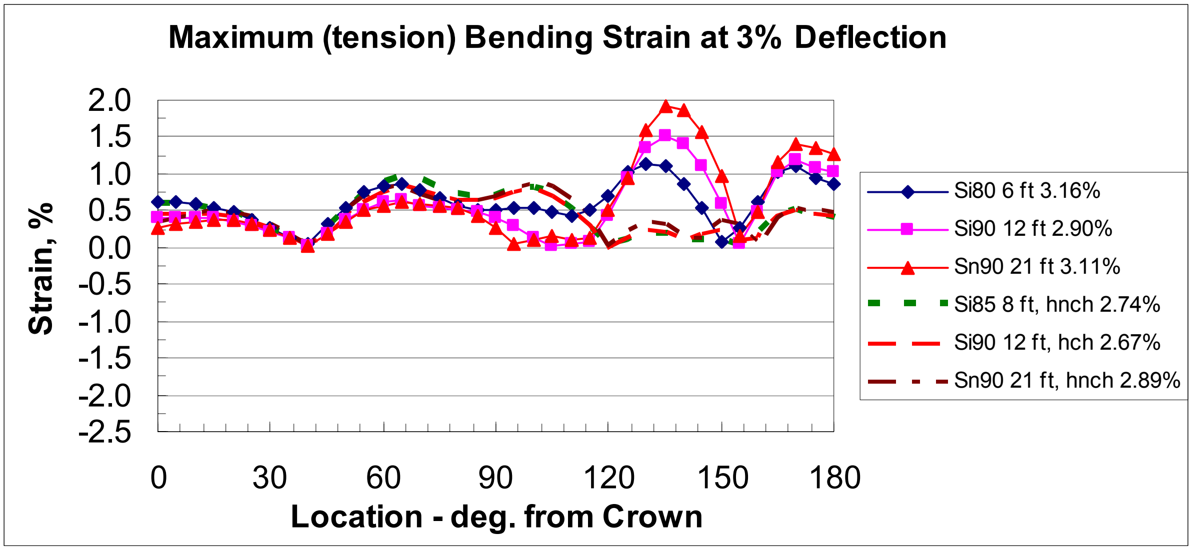

Figure 2.6 – Comparison of Haunched and Unhaunched Bending Strains

2.4.2 Hoop Compression Strain

Fig. 2.7 shows the hoop compression strain for the three backfill conditions at 3% deflection. In this section, and subsequent sections, the deflection considered is the total deflection, that is, the sum of deflection due to bending and hoop compression. Fig. 2.8 makes the same comparison for the Si90 and Si85 backfill at 5% deflection (the Sn90 backfill condition did not reach 5% deflection at a depth of 21 ft. The figures show a variation in the axial strain around the circumference, high at the springline and low at the crown.

The figures also show that the simplified method, with the modifications noted above, gives a reasonable estimate of the minimum hoop stress around the circumference.

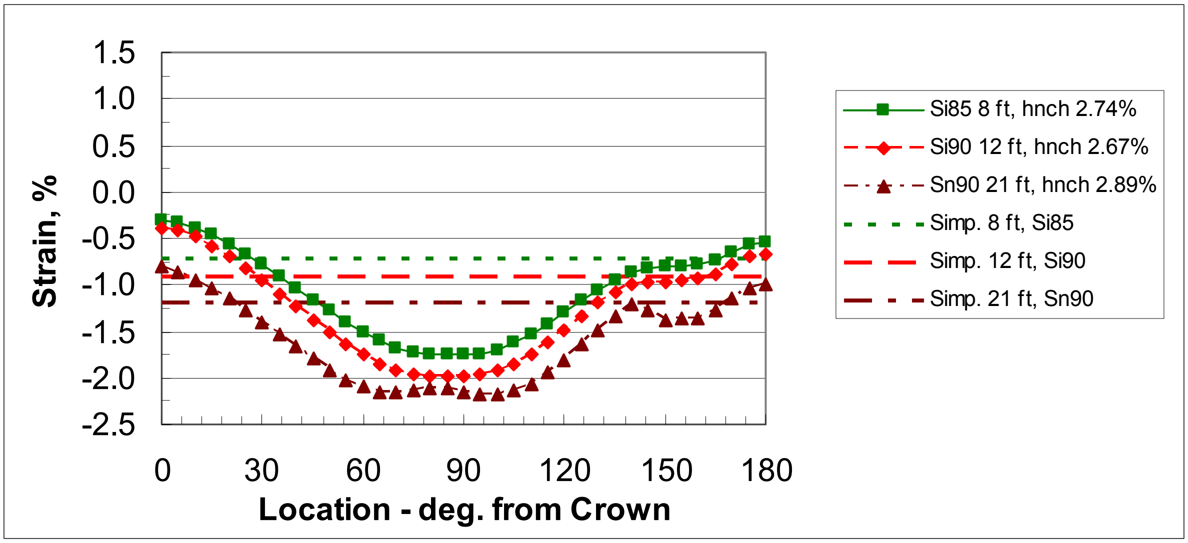

Fig. 2.7 – Axial Strain at 3% Deflection

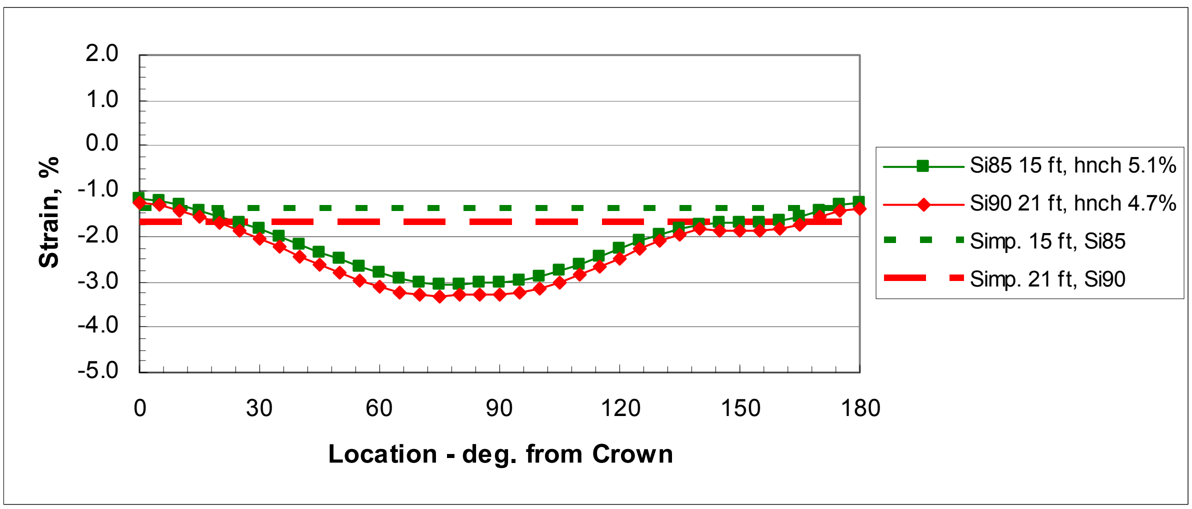

Figure 2.8 – Axial Strain at 5% Deflection

2.4.3 Bending Strain

Figs. 2.9 and 2.10 show the bending strain for the haunched pipes at 3% and 5% deflection respectively. The figure shows that the simplified design procedures provide reasonable estimates of the maximum bending strains.

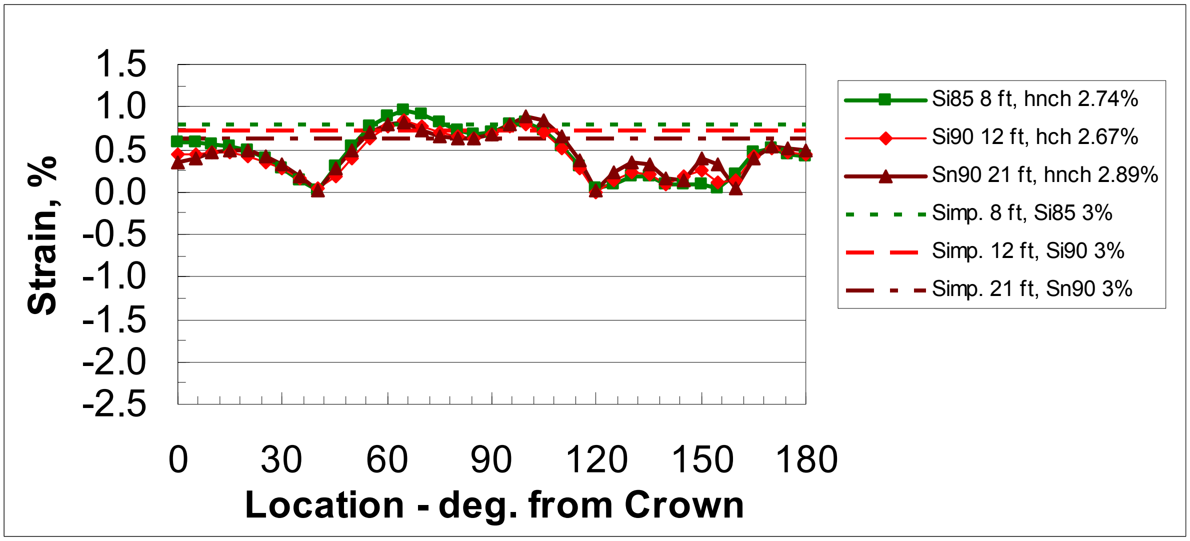

Figure 2.9 – Bending Strain at 5% Deflection

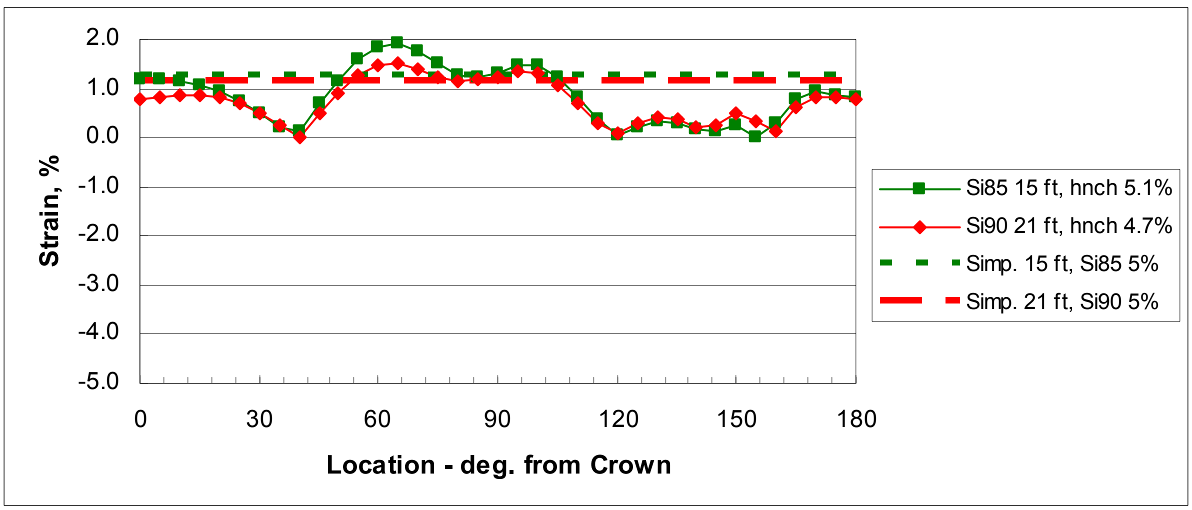

Figure 2.10 – Bending Strain at 5% Deflection

2.4.4 Combined Strain

Figs. 2.11 and 2.12 compare the simplified predictions with the FEA results for total combined strain. The comparison suggests that the simplified procedures can be used to predict total pipe strains for the purpose of estimating the demand on the PE material in service.

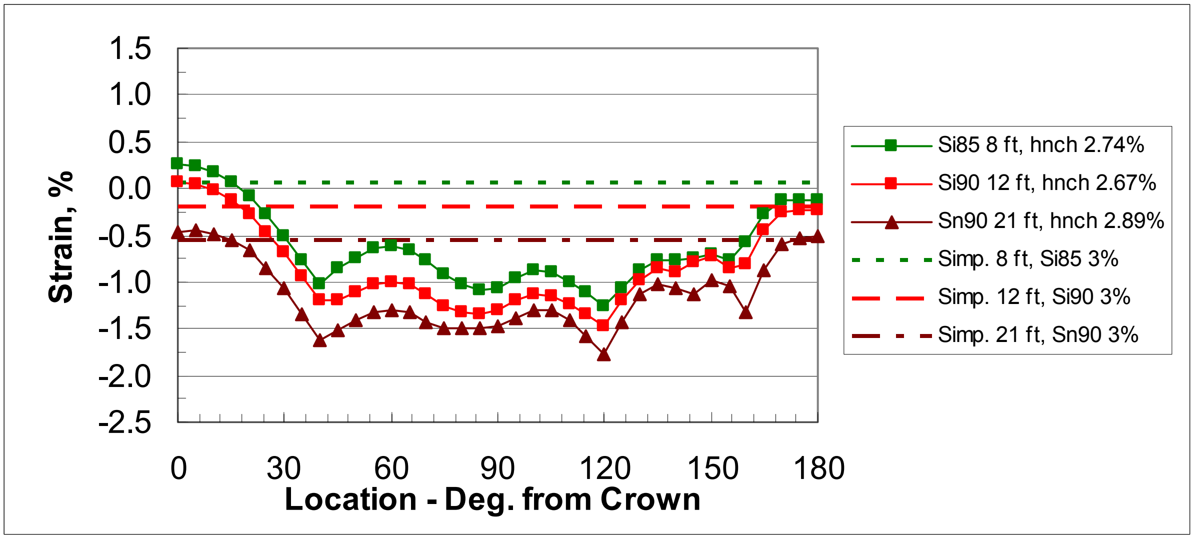

Figure 2.11 – Maximum Combined Strain at 3% Deflection

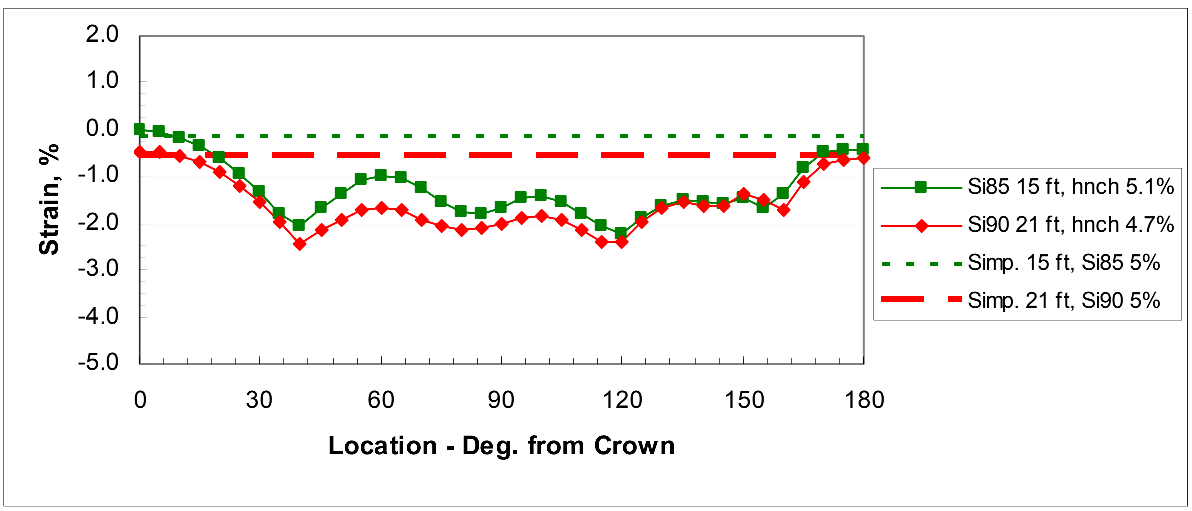

Figure 2.12 – Combined Strain at 5% Deflection

2.4.5 Discussion of Results

The pipe performance calculations presented in the previous section show that tensile strains at limiting deflection conditions are low when the deflection is caused by earth load. Further, as the deflection increases, the tensile strains drop since the hoop compression strain increases at a faster rate than the bending strain, even for the relatively soft Si85 condition. This indicates that the maximum tension strains in corrugated HDPE pipe will likely occur in shallow buried pipe installed at limiting deflection. There is a bit of a conflict in this finding, since:

- if good construction practices are applied, high deflections should not occur at low depths of fill; however,

- construction control procedures for thermoplastic culverts are generally written to limit deflection to a 5% change in diameter, regardless of the depth of fill

The first bulleted item suggests that tension should not occur in buried HDPE pipe at any depth of fill, but the second item shows that deflections are not controlled to lower limits under shallow burial. Also, research has shown that final deflections in flexible pipe often are greatly affected by construction practices (Chambers et al. 1980 and Chambers and McGrath, 1981) and can significantly exceed values predicted by pipe-soil interaction theory. The author has personally observed a number of installations with high deflections under very shallow conditions.

The result of this is the conclusion that either of two design situations should be selected:

- design for a limiting deflection (i.e. 5%) at a low depth of fill or
- reduce the allowable deflection of pipe under low fill conditions.

Since the latter solution would be difficult to control in a field environment, designing for a 5% deflection with little or no thrust is a preferred solution. Using the simplified procedures for predicting bending strain, this suggests that the limiting strain for the 42 in. pipe used in the FEA study should be:

$$\varepsilon_b = D_f\, \Delta/D\, (c_{out}/R) = 4\ (0.05)\ (1.91\text{ in.} / 22.0\text{ in.}) = 1.7\%$$

Assuming a 100 year modulus of 20,000 psi this value relates to a stress of 350 psi. The actual value will vary for different profiles as a function of the profile geometry. Overall, since some thrust will always be present in a profile, a strain limit for circumferential effects of 1.5% (equivalent to a long-term stress of 300 psi) appears reasonable at this time. Applying a factor of safety of 1.5 gives strength limits of approximately 500 psi and 2.5% strain.

2.5 Longitudinal Stresses

The simplified calculations and the FEA study address only the circumferential stresses in HDPE pipe. A relationship to stresses in all directions in the profile is required to assess the total stress state and estimate the material performance. Two papers in the literature address longitudinal stress due to earth loads:

1. Moore and Hu (1995) studied the three dimensional response of corrugated HDPE under pure compression and reported a peak longitudinal tensile stress of 58 psi at an applied stress of 5 psi, roughly equivalent to 6 ft of fill. The longitudinal tension stress occurs on the inside surface of the pipe where the liner meets the corrugation and inspection of the overall stress distribution suggests that this tension occurs as a local

bending of the liner. Using simple extrapolation, the peak longitudinal tension stress at depths of 20 ft would be 193 psi.

2. Moore (1995) studied the three-dimensional response of corrugated HDPE pipe in Sn95 and Sn85 (roughly equivalent to Si90 material) structural backfill under deep fills. They reported maximum longitudinal tension of about 200 psi at the springline in Sn85 backfill at depths of about 12 ft. The tension at the crown and invert are much lower. The location of the peak tension, at the intersection of the liner and corrugation, is the same as in the hoop compression test, and local bending again appears to be the cause. This paper presents only combined stresses, thus the separate effects of bending and axial compression in causing longitudinal tension is not clear. Using simple extrapolation, the longitudinal stress at a depth of fill of 20 ft would be about 330 psi.

NCHRP Report 429 (Hsuan and McGrath, 1999) showed that the intersection of the liner and corrugation was the site of a high percentage of cracks in corrugated HDPE pipe; however, they concluded that the cracks initiated on the interior of the corrugation where the profile has a sharp discontinuity, while Moore indicates that the longitudinal stresses at this location are in compression. Also, as a result of NCHRP Report 429, AASHTO improved the crack resistance of HDPE resins, which was intended to reduce the cracking that was observed.

The other source of longitudinal stress is improper installation, resulting in beam bending type loadings. To control this in the field, Florida will require that pipe grade be controlled to 0.5%. This criterion is applied by assuming that the center of a section of pipe is out of line by 0.005 times the pipe length. Corrugated PE is manufactured in 20 ft lengths, so the criterion allows a deflection of 1.2 in. in a 20 ft span. Assuming that the liner forms a straight tube down the center of a pipe, then, at 0.5% grade misalignment the longitudinal pipe strain is approximately 0.75% or 150 psi in the long-term. As this simplification is conservative, and tension stress due to beam bending will be localized, the two effects need not be considered simultaneously. Thus, HDPE material in corrugated pipe should be able to survive long-term longitudinal tensile stresses of about the same magnitude as circumferential stresses, 300 psi for the service condition, equivalent to approximately 1.5% strain, to provide a 100 year service life.

Note that for both longitudinal and circumferential stresses, the peak stresses do not occur at locations subject to the effects of stress concentrations, thus, the general field capacity of the pipe should evaluated against this criterion rather than the strength of the pipe at points of stress concentration. The provisions for control of stress cracking are assumed to provide protection at these locations.

3. CONSTRUCTION CONSIDERATIONS

Achieving 100 year service life in HDPE pipe requires control of tensile stresses, which are directly related to deflection. Deflections are controlled by backfill selection and control of construction practices. The finite element analysis demonstrates the role of backfill type and compaction in controlling deflections, but the failure to use good practices during backfill placement can increase deflections significantly above values predicted by pipe-soil interaction analysis. Since deflections are in fact controlled more by construction practice than by design, it is increasingly becoming practice to place responsibility for control of deflections on the contractor, rather than the designer. The design process demonstrates that a pipe is adequate at a given deflection and the contractor is then responsible for meeting that deflection level. Construction practices that produce good pipe performance with minimal inspection and construction control are desirable. Key considerations in this are:

- backfill material
- placement procedures
- compaction procedures, and
- on wet sites, control of ground water to allow proper placement and compaction of backfill.

This section on discussion of suitable backfill materials is generally applicable to all types of pipe.

3.1 Backfill Selection

Backfill placement procedures normally require density control to provide the desired backfill properties. Thus, it is common practice to speak of soil properties and relate them to a given percentage of maximum density determined in accordance with a standardized laboratory reference test. The most common tests are the standard Proctor test (AASHTO T99, ASTM D 698) and the modified Proctor test (AASHTO T180, ASTM D 1557). The modified Proctor test applies approximately four times more energy to the soil and thus achieves a higher reference density. For simplicity, references to percent of maximum density in this report refer to the standard Proctor test. For buried pipe, the property of actual interest is the soil stiffness. The ability of a given soil at a given density to resist deformation of a buried pipe is the key mechanism in controlling deflection.

Coarse-grained backfill materials with limited fines (material passing a #200 sieve) content have the highest initial stiffness without compaction and reach the highest stiffness with the least energy. The relative amount of energy to achieve a level of stiffness in various soils is presented in Table 2.1 (McGrath et al., 1990) based on tests of soils in compaction molds. The table demonstrates that to achieve a soil modulus (in this case expressed as the empirical modulus of soil reaction, E'), of 1,000 psi, requires 3 times more energy applied to a Si soil than a Sn soil, and 7 times more energy in a Cl soil than a Sn soil. In common tables of soil moduli (Howard, 1996), only the Sn soils reach a modulus of 3,000 psi.

Table 2.1 – Energy Required to Achieve Soil Stiffness (McGrath, et al., 1990)

Soil Type	Modulus of Soil Reaction, E', (psi)			
	400	1,000	2,000	3,000
Coarse grained soils, ≤ 12% fines (AASHTO Sn soils, Note 2)	≤5	10	17	30
Sandy or gravelly fine grained soils, or coarse-grained soils with fines (AASHTO Si soils, Note 2)	25	33	40	≥100
Fine grained soils (AASHTO Cl soils, Note 2)	50	70	≥100	≥100

Notes: 1. Energy expressed as a percentage of the energy specified in AASHTO T99
2. See AASHTO LRFD Table 12.12.2.4.2-1

Table 2.1 and the above discussion demonstrate that the best backfill materials to allow minimum field control are the Sn soils. There are several classes of these materials:

- crushed rock – crushed rock is a created by crushing cobbles and boulders into smaller angular particles. Crushed rock backfills may be uniform (particles fall into a small size range) or graded, and typically have less than 5% fines. Crushed rock backfill generally provides adequate stiffness when dumped and excellent stiffness when subjected to only minimal compaction. Crushed rock generally performs better than the Sn soils, but no suitable data is available to quantify this, it is generally designed as Sn soil. Crushed rock is typically open-graded, and thus steps must be taken to prevent migration of fines if placed next to fine sands and silts.

- pea gravel – pea gravel is the generic name for rounded, uniformly sized stone. Pea gravel flows well into the haunch zone under the pipe and achieves better stiffness than crushed stone when both materials are dumped, but is not as stiff as crushed stone when both materials are compacted. Pea gravel is open-graded, and thus steps, such as the use of a geotextile fabric must be taken to prevent migration of fines if placed next to fine sands and silts.

- sands and gravels – Sands and gravels without fines achieve good densities when dumped and excellent densities when compacted. If placed, spread and compacted in moderate lift thicknesses, excellent pipe support is assured for all typical installations. Sands and gravels may be well-graded or poorly-graded. Poorly-graded gravels may be susceptible to migration of fines. The only exception to this is uniform fine graded sands. These materials, sometimes called "dune sand" behave more like silts than sands, can be difficult to compact, and are sensitive to moisture content. Use of these materials is controlled by specifying that a maximum of 50% of the particle sizes may pass the No. 100 sieve and a maximum of 20% may pass the No. 200 sieve. Sands not meeting these criteria should be treated as Si materials.

One alternative to specifying coarse-grained backfill materials is to specify controlled low strength backfill (CLSM, also called flowable fill). CLSM is a low strength concrete mix with excellent flow characteristics. It has been shown to provide good pipe performance (McGrath, et al., 1999), but is often quite expensive. It is not discussed further here, but should be considered for installations where the additional cost can be justified.

In Florida, where crushed rock is often not available, sands and gravels are likely the most appropriate choice for structural backfill that will provide the greatest assurance of good performance. These materials provide excellent pipe performance when placed and compacted and are less sensitive to poor construction practices than other materials. We suggest that the preferred backfill meet the requirements of GW or SW material (ASTM D 2487) or AASHTO A-1 or A-3 (AASHTO M145) and meet the limitation on fine sand content listed above. Concrete sand meets these requirements and is generally readily available. Soils with fines (Soils in the Si group) provide good service when properly placed and compacted, but are more susceptible to problems if construction procedures are not followed.

3.2 Backfill Placement

There are many standards that provide construction procedures for buried pipe installation. The most common standard for thermoplastic pipe is ASTM D 2321. This standard provides excellent guidance on a wide range of issues. Most of the issues discussed in this section apply to all types of pipe.

Installation features of particular note that should be present in Florida specifications include:

- trenches should be sufficiently wide to allow joining pipe and proper placement and compaction of the backfill; this condition is generally met if the minimum trench width is 1.5 times the pipe outside diameter plus 12 in.; the space between the trench wall and the pipe should not be less than the width of the compaction equipment in use on the project. If the native soils forming the trench wall do not stand without support (this means structural support and does not include support supplied solely for worker safety in trenches), increase the trench width to provide one half diameter width of structural backfill on either side of the pipe.

- bedding under the pipe, for the central one-third of the pipe diameter should be left uncompacted for a depth of 3 in. This will provide a softer cushion to support the pipe and will mitigate the effects of poor haunching,

- material must be worked into the haunch zone of the pipe, this generally cannot be properly accomplished if the pipe is backfilled to the springline on the first lift,

- use of trench boxes in the zone of backfill at the side of the pipe is prohibited unless specific steps are taken to assure that the backfill is not disrupted or left with a void when the trench box is advanced,

- trenches must be free of water when placing and compacting backfill,

- lift thickness must be controlled, especially on larger diameter pipe; while 6 in, lifts are commonly specified, work has been completed to show that 12 in. lifts can produce good results with coarse-grained backfills, provided placement and compaction practices are suitable, and

- inspection of the completed pipe installation, including a deflection check, is imperative. For large projects, it is recommended to conduct a partial inspection after completion of a small portion of the project; this inspection can be used to adjust construction practices if necessary, and will prevent the large-scale problem of discovering a systematic flaw at the end of a project; AASHTO Specifications require thermoplastic pipe diameter to be at least 95% of the nominal diameter at the completion of construction; in addition to deflection, post-construction inspections should evaluate line and grade, joint conditions and evidence of impingement due to rocks or other debris in the backfill close to the pipe.

The structural backfill over the top of the pipe serves both to provide a complete envelope for the pipe, and to protect the pipe from incidental impacts due to rock in the final backfill. AASHTO currently recommends that the structural backfill be continued over the top of the pipe to a depth of 12 in. This practice should be continued.

3.3 Backfill Compaction

As noted in prior sections, backfill material type and the compaction level both contribute to the overall stiffness of the backfill and the support provided to the pipe to prevent deflection. The suggested coarse-grained materials provide good stiffness when dumped and reach excellent stiffness with the application of minimal compactive energy. It may be beneficial to require a minimum number of passes of compaction as well as specifying a minimum density requirement. If the contractor is in the habit of supplying some compaction, then good pipe performance will likely be the result even if the compaction percentage is slightly less than specified. The specified density should not be less than 95% of maximum.

Backfill must be compacted at the springline of all pipes.

3.4 Minimum Cover Depth

Minimum cover under roadways is controlled more by the affect of the pipe on the pavement than by stress or deflection levels in the pipe. Three studies are examined to address this issue.

3.4.1 Phares et al. (1998)

Phares et al. (1998) conducted tests on 36 in. diameter corrugated HDPE pipe with 24 in. of cover. The backfill conditions consisted of:

3. uncompacted native soil at the sides and over the pipe,
4. compacted granular soil at the sides of the pipe (to about 70% of the diameter) and compacted native soil over the pipe, and
5. compacted granular soil at the sides and about 12 in. over the pipe with 12 in of compacted native soil over that.

Tests consisted of loading the pipe with a static load applied using a reaction load frame to a plate of unspecified size. The report notes that bearing failures occurred under the load plate during the loading but do not specify the size of the plate or the load at which bearing failure initiated. The longitudinal strain at failure did not vary significantly, averaging about 0.14% strain at wheel loads varying from 6,900 lb (uncompacted condition) to 17,800 lb (compacted granular material to 12 in. over the pipe). They do not report the nature of the failure as cracking (tension) or buckling (compression). An HS20 live load consists of a 16, 000 lb wheel, thus the researchers conclude that the factor of safety is on the order of 1 at a depth of 2 ft.

The researchers report results that are inconsistent with load tests on full scale pipe with actual vehicles (see the following review of two additional papers), but the acknowledged bearing failure under the load plate is likely having more impact on the pipe performance than the depth of fill. When the soil fails due to excess bearing stresses, it moves out from under the plate and the plate moves closer to the pipe. In these tests, since a steel plate was used, the stresses under the edge of the plate are likely much higher than at the center, further reducing the bearing capacity. In a highway condition, the tire applies a relatively uniform pressure over the load surface. Since live loads increase as a second order function as the depth of cover is reduced, the load applied to the pipe increases rapidly as the plate moves closer to the top of the pipe. Thus, the reported factor of safety of 1 at a depth of 24 in. is likely quite conservative. For the time being, more emphasis should be placed on results of actual vehicular load tests to determine expected performance. Of some concern is the finding of a failure strain of 0.13%

under a short term loading, however, the behavior that defines the end point of the tests is not identified. See Section 3.4.4 for more discussion of this.

3.4.2 Arockiasamy et al. (2002)

Arockiasamy et al. (2002) report on tests conducted for FDOT as part of an overall assessment of culvert pipes. Pipes with 36 in. and 48 in. inside diameters were buried at depths of 0.5, 1.0 and 1.5 diameters. Two backfills were used, both classifying as poorly graded sands with silt (SP-SM per ASTM D 2487). These materials would both be considered as Si soils per current AASHTO specifications for thermoplastic pipe. Live loads were calculated based on an HS-20 truck with additional load to account for impact per AASHTO LRFD. For the 0.5 diameter burial case, this was an axle load of approximately 40,000 lb. Changes in vertical diameter were about 0.2 in. maximum for the depth of 0.5 diameter, 36 in. pipe. Maximum measured longitudinal tensile strains were 0.05% for the same depth. No failures or damages to the pipe were noted.

In simple beam longitudinal tests, the researchers report failure strains on the order of 0.36% to 0.82%. Slightly larger than those reported by Phares but still extremely low. The nature of the pipe behavior at the end of the test is not identified.

The researchers conclude that minimum cover depth below the top of an unpaved road should be no less than 36 in. or one pipe diameter, whichever is smaller based on the measured longitudinal strains of 0.05% and the Phares reported failure strain of 0.14%.

3.4.3 McGrath et al. (2002)

McGrath et al. (2002) provided an interim report on live load testing of 60 in. diameter pipe under depths of fill of 1 and 2 ft over a period of two years. A total of 8 HDPE pipes were installed, along with one concrete and one corrugated steel pipe that were used as references. The study used two backfill materials, a coarse-grained material without fines and a silty sand with about 25% fines. Both backfill materials were compacted to 90% of maximum. The pipes were installed in the Minnesota Research Road facility, a closed loop road that is subjected to repetitive cycles of truck loads with axle loads of 18,000 and 24,000 lbs. The peak circumferential tensile strains recorded during live load testing were approximately 0.12% at 1 ft of cover. Peak deflections under live load were on the order of 0.12 in. In the cited paper, there was some concern that the deflections were increasing with time; however, continued observation (not yet published) did not bear this out. The deflections increase slightly during the spring thaw but then return to lower values. The overall pipe deflections have been stable for the 2 year life of the project. The testing work is being used to calibrate full three-dimensional pipe-soil models of the live load condition, and the models are then being extended to evaluate design axle loads with impact. These studies, while not published suggest good pipe behavior at a depth of 2 ft.

The Minnesota test did show that the thermal expansion and contraction of the pipe during seasonal changes are significant. The thermal expansion of polyethylene is approximately 10 times that of steel, and, since the backfill at the sides of the pipes is stiff, all of the thermal expansion and contraction is seen as up and down motion of the crown. This led to low spots developing in the roadway during the winter months when the pipe contracted, and also led to cracking in the pavement over the pipes. The cracking was significantly reduced for the two foot depth of cover condition relative to the one foot depth of cover condition. There was some cracking over the crown of the metal pipe and none over the crown of the concrete pipe. Although not final at this time, the researchers are anticipating recommending a minimum cover limit of 2 ft or 0.5 diameters, whichever is greater.

3.4.4 Discussion

Overall, the minimum depth of cover recommended by Arockiasamy et al. (2002) seems to depend mostly on the low longitudinal strengths from their own simple beam tests and those reported by Phares et al. (1998). The reported failure strains suggests a stress on the order of a few hundred psi, which is much lower than strengths reported by Dr. Hsuan in this study for tests on the intersection of the liner and corrugation, which is likely the weakest part of the profile. Neither researcher reports if the failures were due to tensile cracking or compressive buckling, but it is likely that the end point of the tests is the result of a compressive buckling in the pipe wall, a behavior that would be restrained in buried pipe. The reported failure strains from the simple beam tests should not be considered a material limit. Overall, this suggests that The recommended minimum depth of cover of 36 in. or one pipe diameter, whichever is less proposed by Arockiasamy et al. (2002) may be unnecessarily conservative.

At the current time, the minimum depth recommendation from the Minnesota study is recommended as a suitable control to provide good service for unpaved roads. Recommended minimum depths of cover in a format consistent with current Florida specification formats are presented in Table 3.1

Table 3.1 – Recommended Minimum Depth of Cover (in.)

Pipe Diameter	Rigid Pavement Depth below bottom of pavement, in.	Flexible Pavement Depth below bottom of base, in.	No Pavement	
			Commercial	Non-Commercial
up to 48 in.	9	15	24	12
54 in., 60 in	15	21	30	24

For unpaved roads, non-commercial traffic is considered to include applications such as driveway culverts where the typical vehicular traffic does not any include trucks, thus, loading with a vehicle such as an HS-20 truck would be rare. Installation quality, backfill material and backfill compaction are still considered to meet the standards set for other applications.

4. CONCLUSIONS

Florida DOT initiated a study to determine requirements for assuring that corrugated HDPE pipe will provide a 100 year service life. The study was initiated at Drexel University and at Simpson Gumpertz & Heger Inc. Drexel University has prepared a separate report on material strength characteristics that are required to assure good material performance. This report presented the results of the Simpson Gumpertz & Heger Inc. study to evaluate the anticipated stress levels that a pipe installed for 100 years would be subjected to, and recommendations for backfill materials and construction procedures to control stresses. Recommended design and installation procedures are presented in AASHTO format as an attachment to this report.

The study consisted of a parametric analysis of expected performance of buried corrugated HDPE pipe under earth loads with several compaction conditions, varying depths of fill, and variable support under the pipe haunches. This study demonstrates that tensile stresses are relatively low when pipe installation meets typical requirements and maintain changes in vertical diameter to less than 5%. Long-term tensile strain for the service condition should be less than 1.6%, corresponding to a long-term stress of approximately 300 psi, or about 2.5% and 500 psi for the factored load condition. This is significantly reduced from the current AASHTO requirement of 5% long-term tensile strain capacity. Review of a study on three-dimensional analysis of longitudinal strains, and consideration of poor grade control of pipe during installation indicates that this recommended minimum stress level should also apply to longitudinal stresses. In both cases, the limiting stress condition applies to the general field stresses, and is not associated with areas of stress concentration, such as the intersection of the liner and corrugation.

Backfill materials that provide the best performance with minimal controls on construction procedures are well-graded coarse-grained soils (sands and gravels, GW and SW per ASTM D 2487) with less than 12% fines. Uniformly graded coarse-grained soils (GP and SP per ASTM D 2487) also provide good service but are not recommended unless provisions are made to evaluate and control possible migration of fines into open voids. Uniform fine sands should be avoided and criteria were presented for controlling this. Coarse-grained soils with fines (GC, GM, SP, SM, or AASHTO A-2-4 or A-2-5) or fine grained soils with at least 30% coarse grained material (sandy silts and sandy clays) provide good service if placed and compacted properly, but increased inspection during construction is recommended. Backfill should be compacted to at least 95% of maximum standard Proctor density.

The most important aspect of construction control is to inspect corrugated PE pipe after installation, including measuring vertical diameters. Total reduction in vertical diameter should be limited to 5%. On large projects deflections should be evaluated after a small portion of the project has been completed to determine if the construction procedures are adequate.

Suggested minimum cover depths for applications subjected to live loads are based on the findings of the Minnesota study. The minimum depth of fill should be the larger of 2 ft or one-half diameter. Specific recommendations consistent with Florida specifications were presented.

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APPENDIX A

Example of Simplified Calculation Procedures

INPUT

Pipe Properties

Nominal inside diameter .... $D_i := 42 \cdot in$

Profile height............ $ph := 2.925in$

Depth from ouside surface to centroid............ $c_{out} := 1.91in$

Outside diameter............ $D_o := D_i + 2 \cdot ph$ $D_o = 47.85\,in$

Diameter to centroid............ $D := 0.5 \cdot D_o - c_{out}$ $D = 22.02\,in$

Effective area (determined in accordance with AASHTO 12.12.3.5)............ $A_{eff} := 0.37 \cdot \frac{in^2}{in}$

Moment of inertia......... .... $I := 0.45 \cdot \frac{in^4}{in}$

Pipe long-term modulus of elasticity.......... .... $E_{100} := 20000psi$

Load and Resistance Factors

Earth Load, thrust............ $\gamma_{ev} := 1.95$

Earth Load, bending............ $\gamma_B := 1.5$

Soil resistance factor............ $\phi_s := 0.9$

Load modification factor............ $\eta_{ev} := 1.0$

SELECT TARGET DEFLECTION AT GIVEN DEPTH OF FILL

Example use 5% Deflection at 1.5 ft

CALCULATE HOOP STRAIN AT GIVEN DEPTH (New FDOT 12.12.3.4.1)

Height of fill over pipe............ $H := 1.5ft$

Density of soil backfill .... $\gamma_{soil} := 120 \cdot pcf$

Soil prism load............ $P_{sp} := \gamma_{soil} \cdot (H + 0.11 \cdot D_o)$ $P_{sp} = 1.6\,psi$

Calculate vertical arching factor

Constrained soil modulus (AASHTO Table 12.12.3.4-1 (assume Sn95 soil).. $M_s := 2090psi$

Radius to centroid .... $R := \left(\frac{D_o}{2}\right) - c_{out}$ $R = 22.015\,in$

Hoop stiffness factor............ AASHTO LRFD Eq. 12.12.3.4-4 $S_H := \frac{\phi_s \cdot M_s \cdot R}{E_{100} \cdot A_{eff}}$ $S_H = 5.6$

Vertical arching factor............ AASHTO LRFD Eq. 12.12.3.4-3 $VAF := 0.76 - 0.71 \cdot \frac{(S_H - 1.17)}{(S_H + 2.92)}$ $VAF = 0.39$

Calculate minimum hoop thrust compression strain

Crown pressure.. $P_F := \eta_{ev} \cdot \gamma_{ev} \cdot VAF \cdot P_{sp}$ $P_F = 1.232\,psi$

Hoop thrust reduction factor for tension.. $K_T := 0.4$

Minimum thrust.. $T_{Lmin} := K_T \cdot P_F \cdot \frac{D_o}{2}$ $T_{Lmin} = 11.79\,\frac{lbf}{in}$

CALCULATE BENDING STRAIN BASED ON TARGET DEFLECTION AND COMPUTED HOOP STRAIN

Total deflection = bending deflection + hoop thrust compression strain

Total deflection = target deflection .... $\Delta_{T.pctg} := 5 \cdot \%$

Assumed shape factor for bending strain for haunched pipe... $D_f := 4.0$

Bending deflection .... $\Delta := \Delta_{T.pctg} - \frac{T_{Lmin}}{A_{eff} \cdot E_{100} \cdot \gamma_{ev}}$ $\Delta = 4.92\,\%$

Assymmetric centroid - check maxium tension strain at outer fiber; c .... $c := c_{out}$

Bending strain.. AASHTO LRFD Eq. 12.12.3.5.4.6-1 $\varepsilon_{bu} := \gamma_B \cdot D_f \cdot \Delta \cdot \left(\frac{c}{R}\right)$ $\varepsilon_{bu} = 2.56\,\%$

CALCULATE TOTAL MAXIMUM TENSION STRAIN IN SECTION

$$\varepsilon_{tu} := \varepsilon_{bu} - \frac{T_{Lmin}}{A_{eff} \cdot E_{100}} \cdot \frac{\gamma_B}{\gamma_{ev}} \qquad \varepsilon_{tu} = 2.44\,\%$$

Limiting tensile strain... $F_{utension} := 2.5\%$

$$TensLim := if\left(F_{utension} \ge \varepsilon_{tu}, \text{"Design OK"}, \text{"Design Fails"}\right) \qquad TensLim = \text{"Design OK"}$$

APPENDIX B

Specifications for Design

This Appendix presents modifications to the AASHTO LRFD design specifications necessary to evaluate the limiting tensile stress in thermoplastic pipe.

SPECIFICATIONS

12.4.1.3 ENVELOPE BACKFILL SOILS (modify existing section)

For thermoplastic pipes A-1, A-2-4, A-2-5, or A-3

For A-1 and A-3 soils, a maximum of 50% of the particle sizes may pass the 0.150 mm (No. 100) sieve and a maximum of 20% may pass the 0.075 mm (No. 200) sieve. If these limits are not met, treat the backfill as and A-2-4 or A-2-5 material.

12.12.3.31 CHEMICAL AND MECHANICAL REQUIREMENTS (add to Existing Section)

Add the following properties:

Corrugated PE Pipe, AASHTO M 294 shall have the following minimum properties at 100 years:

$F_{utension}$ = 500 psi

E = 20,000 psi

where:

$F_{utension}$ = minimum tensile strength at 100 years.

E_{100} = modulus of elasticity at 100 years.

Performance in compression shall be evaluated using the 50 year properties currently in Table 1.

12.12.3.4.1 MINIMUM THRUST (New Section)

Compute the minimum thrust in the pipe wall as:

$$T_{Lmin} = P_F K_T (D_o/2) \qquad (12.12.3.4.1\text{-}1)$$

where:

K_T = 0.4 for minimum thrust used in calculating

COMMENTARY

C12.4.1.3

A-2-4 and A-2-5 materials require somewhat more care in placing under the haunches, in controlling moisture content and in compacting.

The limitation on A-1 and A-3 soils are set to avoid the use of uniform fine sands. If such materials are used, they are sensitive to moisture content and should be as silty soils.

C12.12.3.31

Minimum properties for 100 year service life are considered temporary until additional testing is completed to determine actual values.

Compression capacity is based on strain. NCHRP Report 438 *LRFD Specifications for Plastic Pipe and Culverts* concluded that the compression strain limit is not time dependent, thus the 50 year limit is applicable to 100 year design.

C12.12.3.4.1

The LRFD specifications have historically focused on calculating the maximum compressive thrust in the pipe wall. Since thermoplastic pipe may crack if subjected to excess tension stress, the minimum thrust stress is required. The maximum

maximum tension force in pipe

In calculating P_F for use in Equation 1, water load and live load shall be neglected.

12.12.3.5.4a General (Modify Existing Section)

Replace Eq. 3 with:

$$\varepsilon_{tu} = \varepsilon_{bu} - \frac{T_{L\,min}}{A_{eff} \quad E_{100}} \text{ x } \frac{\gamma B}{\gamma E} \leq \varepsilon_{tt}$$

(12.12.3.5.4a-3)

redefine ε_{tt} as:

ε_{tt} = factored long term tensile strain

= $F_{utension}/E_{100}$

In Eq. 3 replace the term T_L with the term T_{LMin} as computed in 12.12.3.4.1, and replace the term E_{50} with the term E_{100}.

tension stress is computed as the maximum tension stress due to bending minus the minimum compression thrust stress.

APPENDIX C

Specifications for Installation

This Appendix supplements AASHTO LRFD Construction Specifications, Chapter 30 for Thermoplastic Pipe to meet the requirements of the Florida DOT design specifications for 100 year service life.

SPECIFICATIONS

30.3.2 Bedding Material and Structural Backfill (Add to existing aection)

For A-1 and A-3 soils, a maximum of 50% of the particle sizes may pass the 0.150 mm (No. 100) sieve and a maximum of 20% may pass the 0.075 mm (No. 200) sieve. If these limits are not met, treat the backfill as and A-2-4 or A-2-5 material.

30.5.2 Trench Widths (Add to existing section)

If the trench walls do not stand without support, then increase the trench width to provide a minimum of 1/2 pipe diameter of structural backfill on either side of the pipe.

Trench width must be sufficient to allow compaction of backfill at the springline elevation without damaging pipe.

When supports such as trench boxes are used, ensure that support of the pipe and its embedment are maintained throughout the installation. Ensure that sheeting is sufficiently tight to prevent washing out of native soil from behind the trench box.

Do not disturb the installed pipe and its embedment when moving trench boxes. Trench boxes should not be used below the top of the pipe zone unless methods approved in advance are used for maintaining the integrity of the embedment material. As supports are moved, any voids left by the trench walls below the top of the pipe zone must be filled with specified structural backfill, compacted per these specifications.

COMMENTARY

C30.3.2

A-1 and A-3 soils not meeting these criteria are uniform fines sands and should be handled like A-2-4 and A-2-5 backfill materials.

C30.5.2

Flexible pipe require soil support at the sides, and unstable trench walls are an indication that a wider trench width is required. This criterion does not refer to trenches for which trench supports are required only for worker safety.

30.5.4 Structural Backfill (Modifiy existing section)

Change 1st sentence of second paragraph to:

A minimum compaction level of 95 percent standard density per AASHTO T 99 shall be achieved. In addition to other requirements, backfill must be compacted at least at the springline level of all pipe with diameter greater than 12 in.

A higher density is required to increase assurance that the structural backfill will be stable for the 100 year design life.

30.5.5 Minimum Cover (modify existing section)

Minimum depth of cover for corrugated polyethylene pipe shall be as required in Table 3.5.5-1a

C30.5.5

Table 3.5.5-1a –Depth of Cover (in.)

Pipe Diameter	Rigid Pavement Depth below bottom of pavement, in.	Flexible Pavement Depth below bottom of base, in.	No Pavement	
			Commercial	Non-Commercial
up to 48 in.	9	15	24	12
54 in., 60 in	15	21	30	24